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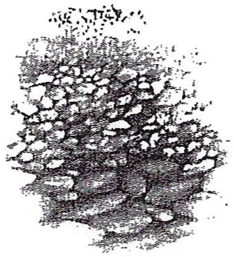
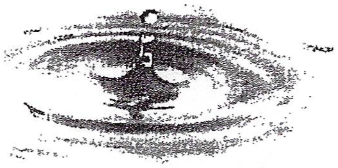


Geotechnical Engineering



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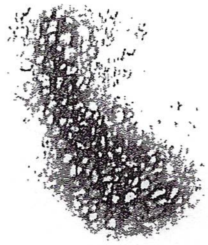
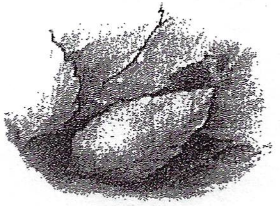


Environmental Assessments and
Remediation



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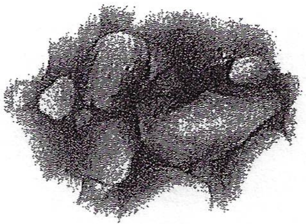

Geologic Assessments

Associated Earth Sciences, Inc.

Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard,
and Geotechnical Engineering Report

GARTZ-HOLT REMODEL

Mercer Island, Washington

Prepared for

Mr. William F. Gartz

Project No. KE070180A
April 17, 2007

Associated Earth Sciences, Inc.
Celebrating 25 Years of Service

April 17, 2007
Project No. KE070180A

Mr. William F. Gartz
7703 West Mercer Way
Mercer Island, Washington 98040

Subject: Subsurface Exploration, Geologic Hazard,
and Geotechnical Engineering Report
Gartz-Holt Remodel
7703 West Mercer Way
Mercer Island, Washington

Dear Mr. Gartz:

We are pleased to present the enclosed copies of the above-referenced report. This report summarizes the results of our subsurface exploration, geologic hazard, and geotechnical engineering studies, and offers recommendations for the design and development of the proposed project.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Bruce L. Blyton, P.E.
Principal Engineer

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND GEOTECHNICAL ENGINEERING REPORT

GARTZ-HOLT REMODEL

Mercer Island, Washington

Prepared for:
Mr. William F. Gartz
7703 West Mercer Way
Mercer Island, Washington 98040

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
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425-827-7701
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April 17, 2007
Project No. KE070180A

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and geotechnical engineering study for the proposed additions to the existing single-family residence. The location of the site is shown on the "Vicinity Map," Figure 1, and the locations of the explorations accomplished for this study and the locations of the proposed additions are presented on the "Site and Exploration Plan," Figure 2. In the event that any changes in the nature, design, or locations of the additions are planned, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be utilized in the design and development of the subject project. Our study included a review of available geologic and past project literature, drilling two exploration borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Geologic hazard evaluations and geotechnical engineering studies were also conducted to determine the type of suitable foundation, allowable bearing pressures, anticipated settlements, lateral earth pressures, floor support recommendations, slope setbacks, and drainage considerations. This report summarizes our current fieldwork and development recommendations based on our present understanding of the project.

1.2 Authorization

Written authorization to proceed with this study was granted by Mr. William F. Gartz. Our study was accomplished in general accordance with our scope of work letter dated March 8, 2007. This report has been prepared for the exclusive use of Mr. William F. Gartz and his agents for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT AND SITE DESCRIPTION

The property is located at 7703 West Mercer Way in Mercer Island, Washington. The existing residence is a two-story structure with a daylight basement that opens up to the rear. The property generally slopes downward to Lake Washington, which lies to the southwest. The

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steep slope between the residence and the lake is broken up by timber walls and is vegetated with landscaping plants. The property is bordered on the northwest and southeast sides by single-family residential properties.

The project consists of constructing two new additions to the south and southwest portions of the existing house, along with the extension of two deck levels and the placement of an architectural "fin" wall at the southwest corner of the existing residence.

In 2001-2002, we performed exploration, design, and construction monitoring work at the property to remediate a shallow slide that took place as the result of a broken sprinkler line. The work performed for this slope remediation project included the design and construction of a Geoweb-reinforced fill zone to improve the stability of the slope. Due to the depth of loose material encountered during the exploration performed as part of this past work, we anticipated that the currently proposed additions will require a deep foundation system, such as pipe piles.

3.0 SITE EXPLORATION

The site exploration was conducted on April 4, 2007, and consisted of two exploration borings and a geologic and geologic hazard reconnaissance to gain information about the site. The various types of materials and sediments encountered in the explorations, as well as the depths where characteristics of these materials changed, are indicated on the exploration boring logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field. If changes occurred between sample intervals in our borings, they were interpreted. The locations of the exploration borings are shown on the "Site and Exploration Plan," Figure 2.

The conclusions and recommendations presented in this report are based on the exploration borings completed for this study. The number, locations, and depths of the explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, interpolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Borings

The two borings were completed on the property using a hand-portable drill rig advancing a 3.75-inch, inside-diameter, hollow-stem auger. During the drilling process, samples were obtained at 2.5- or 5-foot intervals. The borings were continuously observed and logged by an

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engineering geologist from our firm. The exploration logs presented in the Appendix are based on the field logs, drilling action, and inspection of the samples secured.

Upon completion of exploration boring EB-2, the driller was unable to retrieve the lead auger section from the bottom of the borehole. Figure 2 includes measurements taken from the existing residence to the location of this boring.

Disturbed, but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *American Society for Testing and Materials* (ASTM):D 1586. This test and sampling method consists of driving a standard 2-inch, outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 blows are recorded at or before the end of one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. These values are plotted on the attached boring logs.

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification and geotechnical laboratory testing, as necessary.

The various types of soil and ground water elevations, as well as the depths where soil and ground water characteristics changed, are indicated on the exploration boring logs presented in the Appendix of this report. Our explorations and reconnaissance were approximately located by measuring from known site features.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of applicable geologic literature. Our findings are in general agreement with K.G. Troost and A.P. Wisher, 2006, *The Geologic Map of Mercer Island, Washington*, Pacific Northwest Geologic Mapping Project. As shown on the field logs, the borings generally encountered fill overlying dense to very dense pre-Olympia sediments. The following section presents more detailed subsurface information organized from the upper (youngest) to the lower (oldest) sediment types.

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4.1 Stratigraphy

Fill

Fill soils (soils not naturally placed) were encountered at the locations of both exploration borings. Fill encountered at the locations of exploration borings EB-1 and EB-2 generally consisted of loose silty sand with gravel. Portions of the fill at these locations contained organic material. Also, where encountered, the fill differed markedly in thickness, from approximately 4 feet at EB-2 to 15.5 feet at EB-1, at which depth the driller encountered a layer of felt/filter fabric.

The exploration boring performed as part of our past work, approximately positioned at the location of the proposed two-level deck, encountered fill to roughly 17 feet below the ground surface, similar to that encountered at EB-1. Due to their variable density and organic debris content, the existing fill soils are not suitable for foundation support.

Pre-Olympia Fine-Grained Glacial Deposits

Below the fill, our explorations encountered very dense sand/hard silt that extended below the maximum depth explored. The silt was not fractured, although the degree of fracturing can be variable. This fine-grained deposit was interpreted to represent lake or pond deposits placed prior to the Olympia interglaciation and subsequently compacted by the weight of the overlying glacial ice.

This fine-grained material is generally considered suitable for support of light to heavily loaded foundations when in an intact, undisturbed condition. This material is highly moisture-sensitive and susceptible to disturbance when wet.

4.2 Hydrology

Ground water seepage or wet soil cuttings were encountered at both of our exploration borings within the fill zone atop the unweathered native surface and is interpreted to be representative of interflow. Interflow occurs when surface water percolates down through the surficial fill and becomes perched atop the underlying, lower-permeability, unweathered native glacial soils. It should be noted that the occurrence and level of ground water seepage at the site may vary in response to such factors as changes in season, precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic conditions, as observed and discussed herein.

5.0 SLOPE STABILITY ASSESSMENT

The City of Mercer Island geologic hazard maps indicate that the site is located in a landslide hazard area. Therefore, the hazard must be addressed in the design of the foundation. The site's existing slopes are moderately inclined within the existing footprint of the residence, with a steep slope down to the southwest. The near-surface soil underlying the downslope side of the residence site consists primarily of a loose fill zone overlying a very dense/hard, pre-Olympia glacial deposit. We performed a reconnaissance of the site for indications of slope instability, such as bowed or tilted trees, naturally occurring terraced topography, tension cracks, reversed drainage gradients, and unvegetated soil exposures. We did not observe any surface features that would indicate ongoing slope movement on the site or in the immediate vicinity. Also, it is our opinion that the Geoweb-reinforced fill zone constructed in 2002 serves to mitigate the landslide hazard posed by the steep slope behind the residence. Due to the loose nature of the fill soils encountered in our explorations, it is our opinion that the mitigations on the site should include the use of a deep foundation or spread footing placed at an elevation below the encountered fill zone.

6.0 SEISMIC HAZARDS AND MITIGATION

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these events are small and are usually not felt by people. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history. Evaluation of return rates indicates that an earthquake of a magnitude between 6.0 and 7.0 is likely within a given 25- to 40-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

The project site is located within the Seattle Fault Zone. Recent studies by the United States Geological Survey (USGS) (Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and*

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Seattle Basin, Washington, Geology, v. 22, p.71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington-Implications for Earthquake Hazards,* Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) suggest that a northern trace of the east-west trending Seattle Fault (a thrust fault zone) may show evidence of surficial ground rupture. The recognition of the Seattle Fault is relatively new, and data pertaining to it are limited, with the studies still ongoing. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island.

The recurrence intervals for movement along this fault system are still unknown, although they are hypothesized to be in excess of several thousand years. Due to the suspected long recurrence intervals, the potential for surficial ground rupture is considered to be low during the expected life of the structure, and no mitigation efforts beyond complying with the current (2003) *International Building Code* (IBC) are recommended.

6.2 Seismically Induced Landslides

Due to the loose fill materials found during our exploration, the field and subsurface observations noted in Section 5.0, and the very dense characteristics of the native soils underlying this fill, it is our opinion that the risk of seismically induced landslides is low to moderate and predominantly within the upper fill soil sequence. Therefore, as noted previously, we recommend the use of a deep foundation using pipe piles or a spread footing placed at an elevation below the encountered fill zone to mitigate the potential risk.

6.3 Liquefaction

The encountered stratigraphy has a low potential for liquefaction due to the grain-size distribution of the native sediments and the density of the underlying glacially consolidated pre-Olympia sediments. Therefore, no liquefaction mitigation efforts are needed.

6.4 Ground Motion

Based on the site stratigraphy and visual reconnaissance of the site, it is our opinion that any earthquake damage to the proposed structure, when founded on a suitable bearing stratum, would be caused by the intensity and acceleration associated with the event and not any of the above-discussed impacts. Structural design of a building should follow 2003 IBC standards using Site Class "D", as defined in Table 1615.1.1. The 2003 IBC seismic design parameters for short period (S_S) and 1-second period (S_1) spectral acceleration values were determined by the latitude and longitude of the project site using the USGS National Seismic Hazard Mapping Project website (http://earthquake.usgs.gov/hazmaps/). Based on the more current 2002 data, the USGS website interpolated ground motions at the project site to be 1.50g and 0.52g for

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building periods of 0.2 and 1.0 seconds, respectively, with a 2 percent chance of exceedence in 50 years.

7.0 EROSION HAZARDS AND MITIGATION

The City of Mercer Island erosion hazard maps indicate that the site is located in an erosion hazard area. Therefore, the hazard must be addressed in the development of the site. The primary area of concern for erosion hazards on this property is the steep slope on the southwestern portion of the property. Due to the steepness and the slope length, the erosion-related hazard potential is considered to be moderate. It is our opinion that the native vegetation and ground cover on this slope should not be removed or altered.

Apart from the steep slope to the southwest of the existing residence, the erosion-related hazard potential is considered to be low, and special mitigation will not be required beyond the implementation of a Temporary Erosion and Sedimentation Control (TESC) Plan. This plan and a Storm Water Pollution Prevention Plan (SWPPP) will more than likely be conditions of the National Pollutant Discharge Elimination System (NPDES) construction permit. TESC requirements vary between the wet season and the dry season. Between November 1st and April 1st, soil that is to be undisturbed for more than 24 hours is typically required to have temporary cover applied. Drainage control also needs to be established on-site to route turbid runoff to sediment traps or a treatment facility, and to prevent turbid runoff from flowing onto adjacent properties or to sensitive receiving waters. To provide temporary cover, straw mulch, plastic sheeting, or erosion control blankets are typically used. When soil needs to be covered for a longer period of time, temporary seeding can be implemented. Earthwork operations may need to be limited or stopped during periods of heavy rainfall and inclement weather. Upon request, Associated Earth Sciences, Inc. (AESI) can recommend which best management practices (BMPs) should be used in the TESC Plan, help field-fit the BMPs selected for maximum effectiveness, and perform field inspections to assess BMP performance and to provide maintenance recommendations. These field inspections may be required by the Washington State Department of Ecology (Ecology) or the City of Mercer Island for TESC compliance. AESI is also available to prepare a turbidity monitoring plan, if required.

8.0 STATEMENT OF RISK

For Section 19.07.020(E) of the *Mercer Island Unified Land Development Code* (ULDC), the City of Mercer Island requires a statement of risk by the geotechnical engineer. It is AESI's opinion that the development practices proposed for the alteration would render the development as safe as if it were not located in a geologic hazard area, provided the recommendations in this report are followed.

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III. DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the property is suitable for the proposed development, provided the risks discussed are accepted and the recommendations contained herein are properly followed. The bearing stratum of pre-Olympia sediments were encountered at a depth of approximately 4 and 15.5 feet in our explorations and will provide suitable support for steel pipe piles. Conventional spread footing foundations constructed to bear on medium dense to dense native sediment could be utilized to provide foundation support for the proposed addition at the south side of the existing residence.

In consideration of the depth to medium dense sediments and the shallow ground water seepage observed at the time of our exploration, it is our opinion that overexcavation and site preparation for conventional footings would not be feasible for the planned southwest addition, architectural fin, or deck extensions. A deep foundation system, such as piles, will be necessary to reach bearing soil below existing loose fill at the locations of these planned improvements.

10.0 SITE PREPARATION

Site preparation of areas where structural fill is required for future structures should include removal of all trees, brush, debris, and any other deleterious material. Where present, the upper organic topsoil should be removed and the remaining roots grubbed. Foundation areas may then be excavated. Excavation spoils must not be placed on the site slopes.

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, we anticipate that temporary, unsupported cut slopes in the loose fill sediments may be planned at a maximum slope of 1.5H:IV (Horizontal:Vertical). Temporary slopes in pre-Olympia sediments should be limited to 1H:1V. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

As a standard, permanent slopes in structural fill or cut slopes should not exceed a 2H:1V inclination. Permanent slopes in landscaping fill should be limited to 3H:1V.

The fill material and pre-Olympia sediments encountered in the exploration borings contained a high percentage of fine-grained material, which makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened

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soils should be removed and the area brought to grade with structural fill. Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or asphalt treated base (ATB). AESI can provide field design recommendations for these areas, if needed.

11.0 STRUCTURAL FILL

Structural fill may be necessary to establish desired grades or to backfill around foundations and utilities. All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After stripping and subgrade preparation of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the perimeter footings or pavement edges before sloping down at an angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 72 hours in advance to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather and dry subgrade conditions. The on-site soils generally contained significant amounts of silt and are considered moisture-sensitive. In addition, construction equipment traversing the site when the soils are wet can cause considerable disturbance. If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with

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the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction.

A representative from our firm should inspect the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing frequency.

11.1 Keying and Benching

All structural fill planned to be placed on existing slopes steeper than 20 percent (5H:1V) are required to have a keyway constructed at the toe of the fill body and the slope to be benched prior to placing fill. The keyway should be excavated a minimum of 2 feet down into firm, medium dense to dense, native sediments and be a minimum of 8 feet in width. The width of the benches should be established in the field to fit the contour and gradient of the slope being filled.

12.0 FOUNDATIONS

As previously stated, we recommend the use of steel pipe piles for the planned southwest addition, architectural fin, and deck extensions. For the southern addition planned at the location of exploration boring EB-2, it may be possible to use conventional spread footings bearing on medium dense to very dense native sediments or structural fill. Recommendations for both types of foundations are included in this section, although it is our opinion that pipe piles are best suited for the majority of the conditions encountered.

12.1 Pipe Pile Foundations

Pipe piles consisting of 2-, 3- or 4-inch-diameter, driven steel pipe sections will provide suitable support for the proposed residence. The pipe piles should be driven to refusal with equipment appropriate to the pipe diameter. Multiple pipe sections should be joined with compression fittings that fit inside the pipe or welding of the pipe sections. Table 1 summarizes typical wall thicknesses, driving equipment, refusal criteria, and allowable axial compressive loads for each pipe diameter. If higher allowable loads are desired, on-site load testing of at least two piles should be performed to at least 200 percent of the design load to verify that the pile capacities are achievable in the site soils. The load test procedures should be observed by an AESI representative and the test results reviewed by an AESI geotechnical engineer.

Table 1
Pipe Pile Summary

Pile Inside Diameter	Wall Thickness	Typical Installation Equipment	Refusal Criteria* (seconds/inch)	Allowable Axial Compressive Load** (kips)
2-inch	Schedule 80	90-lb pneumatic hammer	60	4
3-inch	Schedule 40	650-lb hydraulic hammer	20	10
4-inch	Schedule 40	850-lb hydraulic hammer	15	16

* Based on listed installation equipment. Other equipment may alter refusal criteria.
** Allowable loads may be increased with acceptable load testing to twice the design load.

If uplift loads are expected to be placed on the piles at any time, the connections should also be securely welded. It should be noted that the uplift capacity of pipe piles is typically not significant and is not used for design. Piles may be battered up to 15 degrees to develop lateral capacity. Battered piles inclined up to 15 degrees should be designed with an allowable axial compressive capacity equal to that used for vertical piles with the axial resistance equal to the horizontal component of the axial load on the pile. Although vertical pipe piles can provide small uplift and lateral capacities, we recommend that these contributions be neglected in designing the new foundation system. Lateral resistance at the foundation level may be provided by passive resistance, as described in the following section. The structural engineer should provide pile spacing, locations, splicing details, foundation connection details, and any other structural design recommendations that are needed. No minimum pile spacing requirements are necessary for pipe piles from a geotechnical standpoint.

Since pipe piles are driven until specific refusal criteria are achieved, rather than to a specific depth, accurate estimation of pile lengths is not possible. Pile lengths will likely be in the range of 10 to 20 feet, but may vary significantly from this range. We recommend that AESI be retained to observe pile installation to confirm that our recommendations have been implemented, to verify that appropriate installation procedures are used, and that the appropriate refusal criteria are achieved. The City of Mercer Island may require this inspection as a condition of permitting.

Passive Resistance

Grade beams and pile caps that are backfilled with structural fill may be designed for passive resistance against lateral translation using an equivalent fluid equal to 200 pounds per cubic foot (pcf). The passive resistance value includes a factor of safety equal to 1.5 in order to reduce the amount of movement necessary to generate passive resistance.

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12.2 Spread Footings

Spread footings may be used for building support when founded on medium dense native soils, as previously discussed. We recommend that an allowable bearing pressure of 2,000 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection; interior footings require only 12 inches burial. However, all footings must penetrate to the prescribed bearing stratum, and no footing should be founded in or above loose, organic, or existing fill soils.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edges of steps or cuts in the bearing soils.

Anticipated settlement of footings founded on medium dense native soils or approved structural fill should be less than 1 inch. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms with the recommendations contained in this report. Such inspections may be required by the governing municipality. Perimeter footing drains should be provided, as discussed under the "Drainage Considerations" section of this report.

13.0 LATERAL WALL PRESSURES

All backfill behind walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 40 pcf. Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid of 55 pcf. Walls with sloping backfill at a maximum angle of 2H:1V should be designed for 55 pcf for yielding conditions and 75 pcf for restrained conditions. If parking areas are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of on-site sandy silts compacted to 92 percent of ASTM:D 1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the wall. Surcharges from adjacent footings, heavy construction equipment, or sloping ground must be

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added to the above values. It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. Perimeter footing drains should be provided for all retaining walls, as discussed under the "Drainage Considerations" section of this report.

13.1 Passive Resistance and Friction Factor

Retaining wall footings/keyways cast directly against undisturbed dense soils in a trench may be designed for passive resistance against lateral translation using an allowable equivalent fluid equal to 200 pcf. The passive equivalent fluid pressure diagram begins at the top of the footing; however, total lateral resistance should be summed only over the depth of the actual key.

The allowable friction coefficient for footings cast directly on undisturbed dense soils may be taken as 0.30. Since it will be difficult to excavate these soils without disturbance, the soil under the footings must be recompacted to 95 percent of the above-mentioned standard for this value to apply.

14.0 FLOOR SUPPORT

Slab-on-grade floors should be constructed to bear on structural fill or pre-rolled, medium dense, native soil. The floors should be cast atop a minimum of 4 inches of washed pea gravel or washed crushed rock to act as a capillary break where moisture migration through the slabs is to be controlled. The capillary break material should be overlain by a 10-mil-thick vapor barrier material prior to concrete placement. American Concrete Institute (ACI) recommendations should be followed for all concrete placement.

15.0 DRAINAGE CONSIDERATIONS

All retaining and perimeter footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel or drain rock. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing and should be constructed with sufficient gradient to allow gravity discharge away from the building. In addition, all retaining walls should be lined with a minimum, 12-inch-thick, washed gravel blanket provided over the full height of the wall that ties into the footing drain. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain. In planning, exterior grades adjacent to walls should be sloped downward away from the structure to achieve surface drainage. All collected runoff must be tightlined to a City-approved location below the steep slopes on the property.

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16.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

At the time of this report, site grading, structural plans, and construction methods have not been finalized. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundation depends on proper site preparation and construction procedures. These inspections may be required by the City of Mercer Island as a part of the building permit conditions. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of this current scope of work. If these services are desired, please let us know, and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident that these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

BRUCE BLYTON
26890
4/18/07

EXPIRES 5/3/08

Jeffrey P. Laub, P.G., P.E.G.
Project Engineering Geologist

Bruce L. Blyton, P.E.
Principal Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs

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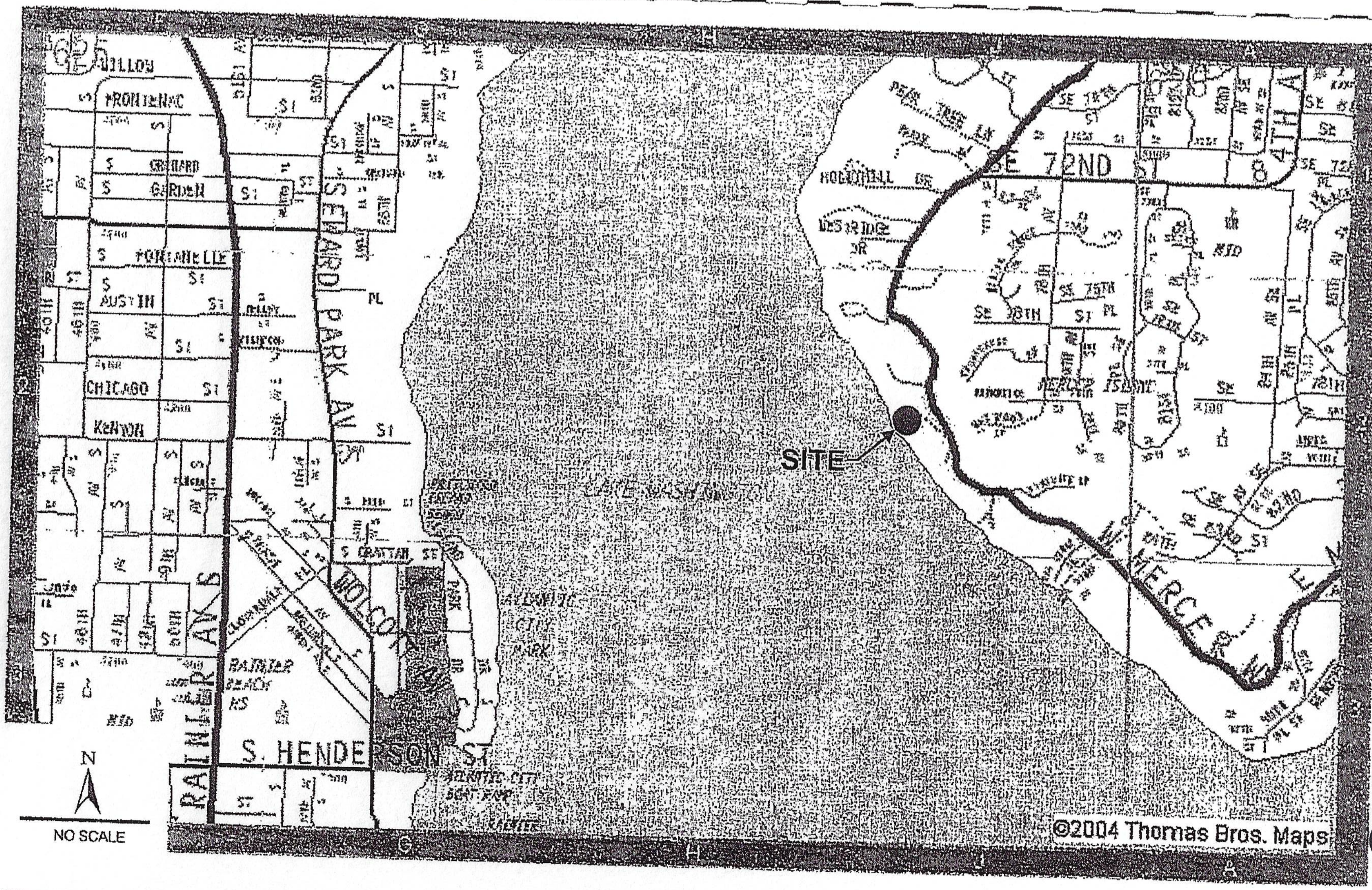

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VICINITY MAP
GARTZ - HOLT REMODEL
MERCER ISLAND, WASHINGTON

FIGURE 1
DATE 4/07
PROJ. NO. KE070180A

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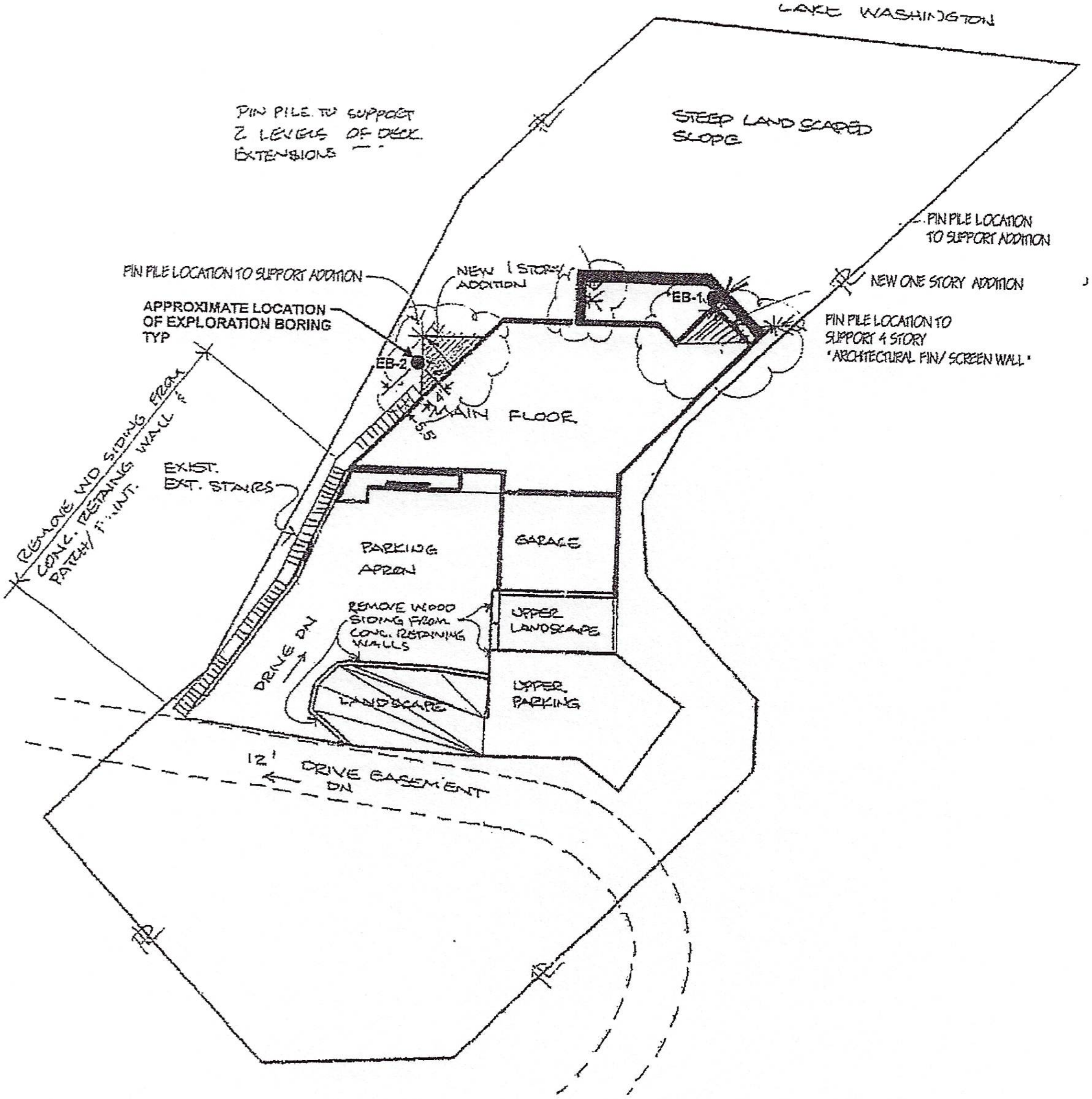

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APPROXIMATE SCALE IN FEET

070180 Gartz - Holt Remodel\070180 Site and Expl.cdr

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SITE AND EXPLORATION PLAN
GARTZ - HOLT REMODEL
MERCER ISLAND, WASHINGTON

FIGURE 2

DATE 4/07

PROJ. NO. KE070180A

APPENDIX

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Exploration Log

Project Number	Exploration Number	Sheet
KE070180A	EB-2	1 of 1

Project Name: Gartz-Holt Remodel
Location: Mercer Island, WA
Driller/Equipment: Boretec/Acker
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): ~62 feet
Datum: MSL
Date Start/Finish: 4/4/07 4/4/07
Hole Diameter (in): 5 1/4 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot 10 20 30 40	Other Tests
				Fill 2-inch thick patio pavers then fill. Hand dig to 4 feet, loose to medium dense, moist, silty SAND, little gravel. At ~2 feet: 12- to 18-inch boulder. Slight seepage 4 to 5 feet.					
5		S-1		**Pre-Olympia Fine-Grained Glacial Deposits** Very dense, moist to very moist, olive, faintly bedded SILT, trace fine sand with thin (<1 inch) interbeds of iron oxide stained, fine SAND, trace silt, trace gravel, dropstones throughout. Firm drill action, occasional rock 5 to 10 feet.			17 30 46	▲76	
10		S-2		**Pre-Olympia Fine-Grained Glacial Deposits** Very dense, moist, olive, massive, silty SAND, few fine to coarse, rounded to subrounded gravel.			50/4"	▲50/4"	
15		S-3		At ~14.5 feet: Cobble based on drill action as at 10 feet.			50/3"	▲50/3"	
20		S-4		**Pre-Olympia Fine-Grained Glacial Deposits** Very dense, very moist to wet, olive, stratified, deformed (ice loading), silty, very fine SAND and SILT, with fine sand laminae, iron oxide stained at ~20 feet.			8 26 50/5"	▲50/5"	
				Bottom of exploration boring at 21.5 feet 4 feet of heave in auger. Last 5 feet of auger was lost at bottom of boring. Boring located at dimensions shown on plan.					
25									
30									
35									

Sampler Type (ST):

- ▯ 2" OD Split Spoon Sampler (SPT)
- ▯ 3" OD Split Spoon Sampler (D & M)
- ☑ Grab Sample
- ☐ No Recovery
- ▮ Ring Sample
- ▨ Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: FSM
Approved by:

1106

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Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE070180A	EB-1	1 of 1

Project Name: Gartz-Holt Remodel
Location: Mercer Island, WA
Driller/Equipment: Boretec/Acker
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): ~52 feet
Datum: MSL
Date Start/Finish: 4/4/07,4/4/07
Hole Diameter (in): 5 1/4 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Topsoil					
				Fill					
	▯	S-1		Loose, moist, mottled light olive-brown and gray, non-stratified, silty fine SAND, few subangular to rounded, fine to coarse GRAVEL, trace organics.			6 5 5	▲10	
5	▯	S-2		At 5 feet: Becomes very loose, only recovered trace sample consisting of moderate brown, slightly moist, SAND, few silt, trace fine organics (root hairs).			1 1 1	▲2	
				Very easy drilling 5 to 10 feet, wet cuttings.					
10	▯	S-3		At 10 feet: Becomes loose, wet. At ~11.5 feet: Increased drill resistance.			1 5 3	▲8	
				Fill? At ~13 feet: Begin harder drill action, smooth drill action to 15 feet. Dense, very moist to wet, light brownish gray and yellow-olive, crudely bedded?, fine SAND, few silt and sandy SILT. At ~15.5 feet: Felt/filter fabric.					
15	▯	S-4		**Pre-Olympia Fine-Grained Glacial Deposits** Dense, very moist, brownish olive and yellowish olive, interbedded, stratified, fine SAND, trace to few silt and faintly laminated silt, iron oxide staining throughout, top of silt is at ~16 feet.			19 23 20	▲43	
	▯	S-5		At ~17 feet: Varies to silty fine SAND, few fine gravel, strong iron oxide staining at 17.5 feet, becomes very dense.			15 50/5"	▲50/5"	
				Bottom of exploration boring at 17.5 feet Drill refusal at 16.5 feet.					
20									
25									
30									
35									

Sampler Type (ST):

- ▯ 2" OD Split Spoon Sampler (SPT)
- ▯ 3" OD Split Spoon Sampler (D & M)
- ☒ Grab Sample
- ☐ No Recovery
- ▮ Ring Sample
- ▨ Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: FSM
Approved by:

AESIBOR 070180A.GPJ April 9, 2007